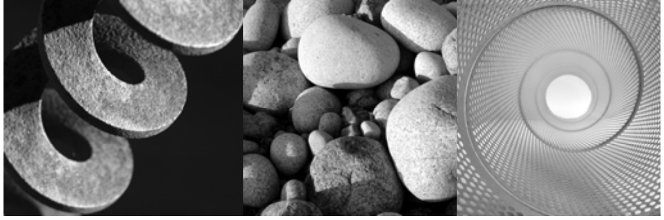

Geotechnical
Environmental and
Water Resources
Engineering

DRAFT

Coal Ash Impoundment – Specific Site Assessment Report

PPL Montana, Colstrip Power Plant

- **Units 1 & 2 Bottom Ash Ponds**
- **Units 1 & 2 "A" Fly Ash Pond**
- **Units 1 & 2 Stage Two Evaporation Pond (STEP)**
- **Units 3 & 4 Effluent Holding Ponds**

Submitted to:
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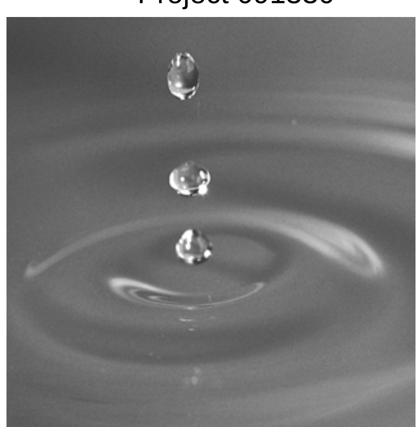
July 2009
Project 091330



Stephen G. Brown, P.E.
Senior Project Manager

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1.0 Introduction

1.1 Purpose

This report presents the results of a specific site assessment of the dam safety of the Units 1 & 2 Bottom Ash Ponds, Units 1 & 2 Fly Ash Pond "A", Units 1 & 2 Stage Two Evaporation Pond (STEP) and Units 3 & 4 Effluent Holding Pond (EHP) coal combustion waste impoundments at PPL Montana's Colstrip Power Plant. These impoundments were assessed because their failure may result in significant economic loss, environmental damage, disruption of lifeline facilities or loss of life (significant or high hazard according to U.S. Environmental Protection Agency (EPA) classification).

1.2 Scope of Work

The scope of work between GEI and Lockheed-Martin Corporation for the site assessment is summarized in the following tasks:

1. Acquire and review existing reports and drawings relating to the safety of the project provided by the EPA and Owners.

2. Conduct detailed physical inspections of the project facilities. While on-site, fill out Field Assessment Check Lists provided by EPA for each management unit being assessed.

3. Review and evaluate stability analyses of the project's coal combustion waste impoundment structures.

4. Review the appropriateness of the inflow design flood (IDF), and adequacy of spillways or ability to store IDF, including considering the hazard potential in light of conditions observed during the inspections or to the downstream channel.

5. Review existing performance monitoring programs and recommend any additional monitoring required.

6. Review existing geologic assessments for the projects.

7. Submit draft and final reports.

1.3 Authorization

GEI Consultants, Inc., performed the coal combustion waste impoundment assessment for the EPA as a subcontractor to Lockheed Martin who is a contractor to the EPA. This work was authorized by Lockheed-Martin under the P.O. No.: 7100052068; EAC #0-381 between Lockheed-Martin and GEI Consultants, Inc. (GEI), dated June 5, 2009.

1.4 Project Personnel

The scope of work for this task order was completed by the following personnel from GEI:

Stephen G. Brown, P.E.	Project Manager/Task Leader
Mary Nodine, P.E.	Staff Geotechnical Engineer

Program Manager for the EPA was Stephen Hoffman. Program Manager for Lockheed-Martin Corporation was Dennis Miller.

1.5 Limitation of Liability

This report summarizes the assessment of dam safety of the identified coal combustion waste impoundments at the Colstrip Power Plant. The purpose of each assessment is to evaluate the structural integrity of the impoundments and provide summaries and recommendations based on engineering judgment. GEI used a professional standard of practice to review, analyze, and apply pertinent data. No warrantees, express or implied, are provided by GEI. Reuse of this report for any other purpose, in part or in whole, is at the sole risk of the user.

1.6 Prior Inspections

PPL Montana personnel indicated that because the Colstrip Power Plant is under a major facilities permit, it is their understanding that the Montana Dam Safety Program rules do not apply to the impoundments on site. PPL Montana regularly conducts informal, internal inspections of their impoundments, including daily drive-by inspections of the downstream toes and the crests of all dams as well as quarterly walking inspections. PPL Montana also voluntarily engages consultants to inspect the dams at approximately 5-year intervals. Inspections were performed in 2005 and in 2009 by Maxim Technologies of Helena, Montana.

2.0 Description of Project Facilities

2.1 General

The Colstrip facility is a coal-fired power plant located in southeastern Montana in the town of Colstrip in Rosebud County (Figure 1). The Colstrip power plant is jointly owned by PPL Montana LLC, a subsidiary of PPL Generation LLC, as well as Puget Sound Energy Inc., Portland General Electric Company, Avista Corporation, PacifiCorp, and North Western Energy LLC. The power plant is composed of four units with a total generating capacity of 2,094 megawatts (MW). Units 1 and 2 began operation in 1975 and 1976 and have capacities of 307 MW each. Units 3 and 4 began operating in 1984 and 1986 and have capacities of 740 MW each. The power plant is located on Arnell's Creek, a tributary of the Yellowstone River.

The Colstrip power plant has numerous on-site waste impoundments including:

- Units 1 & 2 Bottom Ash Ponds;
- Small coal combustion fly ash ponds "A", "B" and "C";
- Units 3 & 4 Scrubber Drain Collection;
- Scrubber Wash Tray; and
- Bottom Ash Ponds.

Of these on-site ponds, only the Units 1 & 2 "A" pond and the Units 1 & 2 Bottom Ash Pond were considered significant hazard impoundments, with the potential for flooding of the town of Colstrip and potential loss of life following a breach. The remaining ponds are not included in this report because they are considered to be low- or less-than-low hazard, either because they have been removed from service, they are incised (all material storage capacity is below grade), the storage capacity is small, and/or their contained waste would not travel any significant distance within or outside the plant in the event of a spill.

In addition to the on-site impoundments, the Colstrip plant has two major impoundments located several miles from the power plant. The Units 1 & 2 Stage Two Evaporation Pond (STEP) is located approximately two miles northwest of the plant, and the Units 3 & 4 Effluent Holding Pond (EHP) is located approximately 3.5 miles southeast of the plant. The Units 1 & 2 STEP was classified as a high hazard impoundment due to the potential for loss of life in the event of a dam breach because of the close proximity of residences within the flood inundation area. The Units 3 & 4 EHP was classified as Low Hazard based on an inundation study (Maxim, 2005). However, GEI recommended the EHP be reclassified as Significant Hazard based on the likelihood of significant economic/environmental cost associated with a dam breach. As a result, the Units 3 & 4 EHP was included in the specific site assessment. An overall view of all onsite and offsite ponds is shown on the aerial photograph (Figure 2).

2.2 Dams and Reservoirs

The Colstrip plant includes several large coal combustion waste dams at the two off-site impoundments, as well as smaller embankments associated with the on-site ponds. The dams included in this report are:

- Units 1 & 2 Bottom Ash Ponds – west, north and east embankments
- Units 1 & 2 “A” Pond – west embankment
- Units 1 & 2 STEP Dam
- Units 3 & 4 EHP
    - Main Dam
    - Saddle Dam

The Units 1 & 2 Bottom Ash Ponds and the Units 1 & 2 “A” Fly Ash Pond are surrounded by a continuous embankment. This earth embankment extends along the west side of the “A” Pond, and continues north to bound the west, north and east sides of the Bottom Ash Ponds. The configuration of these on-site impoundments is shown in the Bottom and Fly Ash Ponds Plan (Figure 3). The embankment has a maximum height of approximately 25 feet, with a 20-foot-wide crest and approximately 2H:1V side slopes. The total length of the embankment is about 4000 feet. Sections of the Bottom and Fly Ash Ponds are shown in Exhibit 1.

The Units 1 & 2 Bottom Ash Ponds are divided into two cells. The east cell stores bottom ash and boiler slag at various stages of clarification, and the water remaining after the ash settles out is transferred to the west clearwell cell. These ponds have a surface area of about 7 acres and a total storage capacity of about 73 acre-feet. The clearwell cell is double-lined with 45 millimeter (mm) reinforced polypropylene (RFP) liners and a leachate collection system. The east cell is clay-lined.

The Units 1 & 2 “A” Pond is currently used to store clean water from stormwater runoff, though the southern portion of the pond contains a small quantity of fly ash on the bottom covered by a geosynthetic clay liner (GCL) and bottom ash. Prior to 2005, the “A” pond was the western portion of a U-shaped pond that also included the “B” fly ash pond to the east. A bottom ash dike was constructed in 2005 to separate the ponds. At this time, an RFP liner was installed in the “B” pond to prepare it for fly ash storage, while the “A” pond remained clay-lined for the purpose of clearwater storage. The “A” pond has a surface area of about 14 acres and a storage capacity of about 245 acre-feet.

The Units 1 & 2 STEP has a total surface area of 176 acres and a total storage capacity of about 4370 acre-feet at the normal operating pool of El. 3270. The pond is divided into five cells as shown in the plan on Figure 4. Three of the cells are currently in use. All cells have single high density polyethylene (HDPE) liners with the exception of Area “B”, which has a double, 45 mil reinforced polyethylene (RFP) liner with leak detection and leachate collection systems. The STEP currently stores fly ash, bottom ash, boiler slag, flue gas residuals and mill rejects. The coal combustion waste is pumped into the pond as a slurry, and the water is decanted and pumped back to the Colstrip plant for reuse. The remaining fly ash solidifies as evaporation occurs.

The Units 1 & 2 STEP Dam was constructed in 1992 and is 2400 feet long with a maximum height of 88 feet, a 25-foot-wide crest and 3H:1V side slopes. The dam crest is at El. 3278, providing 8 feet of free board above the normal pool elevation. The dam is constructed of earth fill and has a zoned cross section with a central core extending to bedrock in a core trench. The dam also features a grout curtain extending up to 80 feet below the core trench for seepage control. An upstream low-permeability soil blanket was constructed on the left abutment area to reduce potential seepage. A chimney drain, blanket drain and toe drain collect and control seepage that moves through the dam. A valley drain system collects surface water, groundwater, and potential seepage and returns it to the ponds. The STEP Dam is located about 3000 feet downstream of the Stage One Evaporation Pond and its associated dam, which was completed in 1977 and has been completely filled with coal combustion waste. The area has since been reclaimed and is currently used as pasture land. A plan and profile of the STEP Dam is shown in Exhibit 2, and typical sections are shown in Exhibit 3.

The Units 3 & 4 EHP has a planned total surface area of 367 acres and a storage capacity of about 17,000 acre-feet at the normal operating pool of El. 3280. The pond is divided into eight cells storing plant coal combustion waste including fly ash, bottom ash, boiler slag, flue gas residuals, mill residuals, and cleaning chemicals. The General Plan for the Units 3 & 4 EHP is shown in Figure 5. The coal combustion waste in the Units 3 & 4 EHP is currently stored as paste with 68 percent solids, made by an on-site paste plant, though prior to construction of the paste plant several years ago the waste was stored as traditional slurry with 10 to 15 percent solids. Areas “B” and “F” are double-lined with RFP with a leachate collection system. The remaining cells are not currently lined. The entire pond is surrounded by a concrete cutoff wall and underlain by claystone or siltstone to control seepage. The cutoff wall is up to 80 feet deep and is keyed 5 feet into claystone or siltstone.

The Units 3 & 4 EHP has two dams: the Main Dam and the Saddle Dam, which are about 2300 feet and 3500 feet long, respectively. The dams were constructed in 1983 and currently have crests at El. 3262, though future plans include raising both dams to El. 3290 as described in the design report (Bechtel 1982). The Main Dam has a maximum height of 110 feet and the Saddle Dam has a maximum height of 38 feet. Both dams are operated with greater than ten feet of freeboard. The dams are constructed as zoned rolled earth embankments with central cores extending to bedrock in a core trench. The dams also feature chimney drains, core trench sloping drains, horizontal blanket drains and drainage pipes. The blanket drain is limited in extend to the central third (highest sections) of the dam. A valley drain system downstream of the dams collects surface water, groundwater and potential seepage for pumping back to the reservoir. Plans and profiles of the Main and Saddle Dams are shown in Exhibits 4 and 5. Typical sections and details are shown in Exhibit 6.

Information concerning the dams at the Colstrip facility is presented in Table 1.

Table 1: Colstrip Power Plant - Dam Parameters Summary

Parameter	Value				
Dam	Units 1 & 2 Bottom Ash Pond Embankment	Units 1 & 2 "A" Fly Ash Pond Embankment	Units 1 & 2 STEP Dam	Units 3 & 4 EHP Main Dam*	Units 3 & 4 EHP Saddle Dam*
Height (ft)	25	25	88	110	38
Length (ft)	~2000	~2000	2400	2300	3500
Crest Width (ft)	20	20	20	136	153
Crest Elevation (ft)	3264	3264	3278	3262	3262
Side Slopes	2H:1V	2H:1V	3H:1V	3H:1V	3H:1V
Operating Pool El. (ft)	3260	3260	3270	3237	3237
Normal Storage Volume (ac-ft)	73	38.4	4370	Est. 10,000	
Normal Surface Area (acres)	7	7.6	176	Est. 300	

*Note: Final storage capacity for the Units 3 & 4 EHP will be 17,000 acre-feet and area 367 acres after 28-foot raise to El. 3290. No area-capacity curve was available for the estimate.

2.3 Spillways

The on-site storage ponds do not have spillways. These ponds are designed to have a minimum 4 feet of freeboard, which is considered sufficient to impound the 24-hour probably maximum flood (PMF) (24 inches) and is expected to be sufficient to impound the remaining rainfall which is included in the 72-hour PMF.

The Units 1 & 2 STEP has an emergency spillway at El. 3274.6 which was originally designed to prevent the dam from overtopping in the unlikely event that the 24-hour PMF occurs subsequent to the 100-year flood. The spillway is an uncontrolled open channel spillway excavated into the left abutment (looking downstream) of the embankment. The spillway is approximately 100 feet long and 25 feet wide. The PMF was updated in 1988, when the HMR was revised, to be associated with the 72-hour probable maximum precipitation (PMP) rather than the 24-hour PMP. Routing the 72-hour PMF results in a small portion (50 acre-feet) to be discharged through the emergency spillway.

The Units 3 & 4 EHP does not currently have an emergency spillway, though when the dam is raised to its final height, an emergency spillway at El. 3286.1 is planned to prevent the dam from overtopping should the 24-hour PMF and the 100-year flood occur in succession. The planned spillway consists of a gabion-lined channel with a 50-foot-wide crest. The current dam configuration provides over 20 feet of freeboard and is capable of storing the PMF.

A summary of the spillway parameters is presented in Table 2.

Table 2: Colstrip Power Plant - Spillway Parameters Summary

Parameter	Value	
Reservoir	Units 1 & 2 STEP	Units 3 & 4 EHP (proposed)
Spillway Length (ft)	~200	None
Crest Elevation (ft)	3274.6	None
Crest Width (ft)	25	N/A
Side Slopes	Unknown	N/A

2.4 Intakes and Outlet Works

There are no intake or outlet work structures associated with the ponds at the Colstrip facility. Water levels are controlled by pumping ash slurry or paste into the ponds at a slow rate. Water exits the ponds through evaporation.

2.5 Drains

PPL Montana personnel indicated that the on-site ponds have a system of toe drains that drain into a valley drain system. Water is then collected and pumped back into the ponds.

The Units 1 & 2 STEP Dam has chimney drains, inclined drains and horizontal drainage blankets on the downstream side of the embankments to collect seepage. Seepage collected by the aforementioned drains is directed to the toe drains, which ultimately drain to a valley drain system. The valley drain system consists of a 20-inch drain pipe extending downstream from the toe of the dam along the stream channel. The pipe discharges into a manhole where water is collected and pumped back to the ponds.

The Units 3 & 4 EHP Dams have chimney drains, inclined drains and horizontal drainage blankets on the downstream side of the embankments to collect seepage. Seepage collected by the aforementioned drains is directed to the toe drains, which ultimately drain to a valley drain system similar to that described above for the Units 1 & 2 STEP Dam.

2.6 Vicinity Map

The Colstrip Power Plant is located within Rosebud County, Montana in the city of Colstrip, as shown on Figure 1. The plant is located in the East ½ of Section 34, Township 2 North, Range 41 East. The Units 1 & 2 STEP is located approximately two miles northwest of the plant, and the Units 3 & 4 EHP is located approximately 3.5 miles southeast of the plant. The Units 1 & 2 STEP is located in Section 29, Township 2 North, Range 21 East. The STEP is located in a dissected stream valley draining into Arnell's Creek and ultimately into the Yellowstone River. The Units 3 & 4 EHP is located in Sections 5 and 6, Township 1 North, Range 42 East. The EHP is located on a tributary to Cow Creek.

2.7 Plans and Sectional Drawings

Engineering drawings and reports for various project features are available in the Owner's files. For reference purposes, project plan and sectional drawings from the Owner's files are reproduced in this report as follows:

Bottom and Fly Ash Ponds Sections	Exhibit 1	(Dwg C1-32)
STEP Finished Plan and Profile of Dam	Exhibit 2	(Dwg C1-933)
STEP Typical Sections	Exhibit 3	(Dwg C1-934)
Units 3 & 4 EHP Main Dam Plan and Cross Section	Exhibit 4	(From Chen 1989)
Units 3 & 4 EHP Saddle Dam Plan and Cross Section	Exhibit 5	(From Chen 1989)

2.8 Standard Operational Procedures

The Colstrip facility is a coal fired power plant that provides electric power to millions of customers. The plant is composed of two 307 MW units (1 & 2) and two 740 MW units (3 & 4), with a total generating capacity of 2,094 MW. Coal is delivered to the power plant by trains and conveyor systems, where it is then combusted to power the steam turbines. The burning of coal produces several gases which are vented from the boiler, and bottom ash, which is made of coarse fragments, falls to the bottom of the boiler, and is removed along with boiler slag.

The bottom ash and fly ash from Units 1 and 2 are pumped as slurries to the on-site ponds just south of the plant. Partial settling of particulates occurs in these ponds and the remaining clearwater is returned to the plant. Some of the bottom ash is reclaimed from the on-site ponds and used for construction of roads and dikes on site. The remaining fly ash and bottom ash is pumped to the Units 1 & 2 STEP Pond northwest of the site for final settlement and storage.

The bottom ash from Units 3 & 4 is pumped to the on-site bottom ash ponds to the east of the plant for temporary storage, and ultimately to the Units 3 & 4 EHP Pond to the southeast of the plant for final settlement and storage. The fly ash from Units 3 & 4 is pumped directly to the Units 3 & 4 EHP paste plant prior to being pumped into the pond. The fly ash slurry is processed in the paste plant and pumped into the pond as paste with 68 percent solids (versus 10-15 percent solids for typical scrubber slurry).

3.0 Summary of Construction History and Operation

The power plant is composed of four units with a total generating capacity of 2,094 MW. Units 1 and 2 began operation in 1975 and 1976 and have capacities of 307 MW each. Units 3 and 4 began operating in 1984 and 1986 and have capacities of 740 MW each.

The on-site ponds, including the Units 1 & 2 Bottom Ash Ponds and “A” Pond, were designed at the time Units 1 & 2 were constructed, in the mid-1970s. Original design reports for these ponds and their embankments are not available. In 2005, the “A” Pond was divided by constructing a dike to form an adjacent “B” Pond to the east. The dike was designed by HKM Engineers, and its design and construction is documented in the design report (HKM, 2005).

The Units 1 & 2 “A” Pond was originally used to store fly ash, with slurry from the plant entering the attached “B” Pond at the northeast corner. The water moved south around the bottom of the “U” shape and then north into the “A” pond as the fly ash settled out, leaving some waste at the southern end of the “A” pond but the north end relatively clear. When the ponds were separated by a dike in 2005, the fly ash at the bottom of the southern end of the “A” pond was covered with a GCL and compacted bottom ash for permanent storage. The “A” pond is currently used for stormwater or decant water storage.

The Units 1 & 2 STEP Dam was designed and constructed by Bechtel Engineering in the late 1970s to early 1980s. The embankment is zoned earth fill, with a silt and clay core extending into the sandstone and siltstone bedrock, and a shell consisting of weathered sandstone, siltstone, shale and non-plastic silt. The STEP is downstream of the old Stage One Evaporation Pond, which was completely filled with coal combustion waste and reclaimed in the early 1980s. A paste plant is being constructed at the STEP, with expected completion in 2009. Following completion of the paste plant, all fly ash will be disposed of as paste in the STEP.

The Units 3 & 4 EHP Main Dam and Saddle Dam were designed and constructed by Bechtel Engineering in the early- to mid-1980s. The embankments are zoned earth fill, with low- to medium plasticity silt and clay cores and shells consisting of weathered sandstone, siltstone, shale and non-plastic silt. The dams currently have crests at El. 3262, but plans include ultimately raising the dams 28 feet to El. 3290 and constructing an emergency spillway. In about 2008, PPL began concentrating the fly ash slurry from the typical 15 percent solids to approximately 68 percent solids (termed “paste”). A paste plant was constructed at the EHP and currently fly ash is disposed of as paste or 15 percent solids fly ash in the EHP cells.

4.0 Geologic and Seismic Considerations

The Colstrip Power Plant and its associated impoundments are located in and near the town of Colstrip in Rosebud County, Montana. This area of Montana is within the Northern Great Plains Physiographic Province, which is characterized by valleys, plains, isolated buttes and long, narrow flat-topped ridges. The region contains steep slopes capped by the resistant baked shale, or "clinker", prominent in the area. The baked shale was formed by the burning of underlying coal deposits.

The Colstrip region bedrock is part of the Tongue River Member of the Upper Cretaceous to Paleozoic Fort Union Formation. The Tongue River Member is composed of claystone, shale, siltstone and sandstone with deposits of lignite, coal and calcareous sedimentary rocks. The rocks in this unit generally dip less than a few degrees to the south-southeast.

Seismic acceleration based on the on the Uniform Building Code Seismic Zone Map maximum ground motion for Rosebud County is 0.05g, which corresponds to an earthquake return period of about 2,500 years. This value is consistent with the United States Geological Survey regional probabilistic ground motion associated with a similar return period.

Site-specific documentation presenting geologic information for the facilities at Jeffrey Energy Center included:

- Portage and HKM 2005 "PPL/Colstrip Fly Ash Pond Design and Construction Report"
- Bechtel 1979 "Second Stage Evaporation Pond Design Report"
- Bechtel 1982 "Effluent Holding Pond Design Report"

Borings drilled near the on-site Bottom Ash and Fly Ash Ponds indicate that the stratigraphic section includes about 2 feet of surface fill overlying about 10 to 20 feet of predominantly fine-grained soils. The overburden soils are underlain by hard sandstone or shale and intermittent coal. Geotechnical boring logs and detailed geologic information is not available for the on-site ponds.

Borings drilled during the site investigation for the Units 1 & 2 STEP Dam indicate that the stratigraphic section includes up to 35 feet of clayey silt and gravel overburden overlying a one-foot-thick remnant of the McKay Coal Seam, 60 feet of poorly- to moderately-cemented sandstone and siltstone, 25 feet of shale, and alternating moderately-cemented siltstone and shale, with thin lenses of carbon and limestone throughout.

The Units 3 & 4 EHP Dams are situated in an oval-shaped erosional basin within the Fort Union Formation. Baked shale forms the majority of the rim of the basin while sandstone, siltstone and occasional coal and claystone form most of the basin bottom. A thin veneer of residual silty sand and sandy silt blankets most of the area, ranging in thickness from 1 to 18 feet. Interbedded sandstone, siltstone and shale underlie most of the site. The McKay Coal Seam is present within the deeper bedrock and averages about 10 feet thick in the EHP area. The stratigraphic sections of both the Main Dam and the Saddle Dam consist of the highly permeable baked shale overlying lower-permeability bedrock. The baked shale extends from the dam crest El. 3262 down to about El. 3230 in the abutments of the Main Dam and is found in both abutments and beneath the Saddle Dam to about El. 3210. The presence of the permeable baked shale was addressed in the design by a perimeter concrete cutoff wall for seepage control at the Units 3 & 4 EHP site.

5.0 Instrumentation

5.1 Location and Type

A large network of monitoring wells are installed throughout the Colstrip facilities primarily in support of groundwater quality studies. A few of these wells are located in the abutments or near-downstream area of the dams and serve a dual purpose to also monitor seepage. Only a few instruments are purposely assigned for monitoring the performance of the dams. The wells are monitored monthly. Only partial well location or water level data was available for this report. A line of interceptor wells is located just downstream of the dam to pump seepage and groundwater back to the reservoir for groundwater quality purposes.

5.1.1 Units 1 & 2 Bottom Ash and “A” Pond

There are no piezometers or movement monuments installed in the embankments around the Bottom Ash or “A” Pond. Interceptor wells located downstream of the ponds enable groundwater to the collected and pumped back to the ponds.

5.1.2 Units 1 & 2 STEP Dam

There are no piezometers or movement monuments installed in the STEP Dam embankment or abutments. There are four observation wells installed just downstream of the dam. Water level data was not available for the observation wells located downstream of the dam. Seepage collected by the internal drains and toe drain is discharged into the valley drain trench, which is an approximately 500-foot-long gravel and perforated pipe trench that terminates in a manhole. Seepage from the dam is comingled with surface water and potentially groundwater, therefore the quantity of seepage collected by the internal drain system is unknown.

5.1.3 Units 3 & 4 EHP Main Dam

Prior to 2001, there were no piezometer instruments in the Main Dam or abutments. Two electric piezometers were installed to obtain pore pressure information near the bottom of the core at a location just downstream of the concrete cutoff wall, and four standpipe piezometers in the abutments to observe groundwater conditions; two in the sandstone and two in the baked shale adjacent to the dam (Hydrometrics, 2001). One of the two electric piezometers failed one day after installation and has not been replaced. Locations of these instruments are shown in Appendix A.

Seepage collected by the internal drains and toe drain is discharged into the valley drain trench, which is a gravel and perforated pipe trench that passes through a manhole located near the downstream toe. The flow in the manhole was estimated by eye to be about 20 gallons per minute (gpm). Seepage from the dam can potentially be comingled in the valley drain trench with surface water and shallow groundwater, therefore the quantity of the observed seepage flow that is collected by the internal drain system is unknown.

5.2 Time Versus Reading Graphs of Data

5.2.1 Units 3 & 4 Main Dam

The 2001 report by Hydrometrics and the 2009 report by Womack Associates, Inc. present data spanning a 6-year period for the single remaining electric piezometer installed in the dam core.

Long-term data for the standpipe piezometers installed in the dam abutments is not presented in the reports, but the piezometers installed into the baked shale strata are reported to be dry from the time of their installation in June 2001 until December 2001. The piezometers installed into the sandstone in the dam abutments had water levels at El. 3187 and El. 3188 (75 to 77 feet below the ground surface) in June 2001.

The electronic piezometers were installed in borings 636-P and 637-P, which were drilled to final depths of 119 and 111 feet, respectively. Data from electric piezometer 636-P is tabulated for the period June 2001 to October 2007 and for both piezometers 636-P and 637-P (broken) for the period June 2001 to December 2001 in Appendix A.

5.3 Evaluation

5.3.1 Units 1 & 2 Bottom Ash and "A" Pond

There are no instruments for monitoring the performance of the pond embankments, therefore the instrumentation program is inadequate.

5.3.2 Units 1 & 2 STEP Dam

There are no instruments for monitoring the internal water pressure, movement, or seepage flow rates at this dam, therefore the instrumentation program is inadequate.

5.3.3 Units 3 & 4 Main Dam

There is only one functioning piezometer available to monitor water pressures internal to the dam embankment. There are two piezometers available to monitor water pressures in the sandstone in the dam abutments. The last available reading for the embankment piezometer was in 2007. The last available reading for the abutment standpipe piezometers was in 2001.

Instrumentation was installed in 2001 to evaluate a potentially significant dam safety seepage issue for seepage pressures in the sandstone to go around the cutoff wall and act on the downstream shell of the dam embankment. An instrument that failed shortly after installation has not been repaired or replaced. The number of instruments and the frequency of monitoring is inadequate to develop a full understanding of the pore pressure conditions in the dam core and downstream shell, or to identify changes in conditions over time. Therefore, the instrumentation program is considered inadequate at this dam.

6.0 Field Assessment

6.1 General

Site visits to assess the condition of the Units 1 & 2 Bottom Ash Ponds, “A” Pond and STEP and the Units 3 & 4 EHP at the Colstrip Power Plant were performed on June 2 and 3, 2009 by Stephen G. Brown, P.E., and Mary C. Nodine, P.E., of GEI. Joe Byron of the Environmental Protection Agency, Gordon Criswell and Mike Holzwarth of PPL Montana and Ray Womack, P.E. of Womack Associates (Geotechnical consultant for PPL Montana) assisted in the assessment. Also present was Iver Johnson of the Montana Department of Environmental Protection.

The weather during the site visits was generally overcast with occasional light rain, with the temperatures around 50-60 degrees Fahrenheit. The ground surface was dry on the first day of the inspections (June 2), and it rained overnight prior to the second day (June 3) making the ground surface and vegetation damp.

Field observations are organized as follows:

- Units 1 & 2 Bottom Ash Ponds – west, north and east embankments
- Units 1 & 2 “A” Pond – west embankment
- Units 1 & 2 STEP Dam
- Units 3 & 4 EHP
    - Main Dam
    - Saddle Dam

A checklist is provided in Appendix B and photographs are provided in Appendix C. Sections 6.2 through 6.5 describe observations made during the assessment relative to key project features. Section 6.6 presents specific observations.

6.2 Units 1 & 2 Bottom Ash Ponds

Field assessment of the Units 1 & 2 Bottom Ash Ponds included walking the embankment crest, upstream slope and downstream slope. We saw no obvious signs of settlement or displacement, and only one instance of seepage that should be remedied in order to improve the safety of the impoundment. General photos of the Units 1 & 2 Bottom Ash Ponds are shown in Photos 1 (west cell) and 2 (east cell).

6.2.1 Embankment Crest

The embankment crest appeared to be in good condition. No signs of cracking or settlement were observed during the assessment No vegetation was present on the dam crest. (Photos 3 and 4).

6.2.2 Upstream Slope

The upstream slope of the bottom ash pond embankment is protected from erosion by an RFP liner (west cell – Photo 5) and a clay liner (east cell – Photo 6) and appeared to be in good condition. A concern with regard to the upstream slope was a 24-inch HDPE pipe that protrudes from the interior southwest corner of the west cell of the bottom ash ponds (Photo 7). This pipe serves as a carrier pipe for two smaller 4-inch discharge pipes. The carrier pipe terminates in the interior of the embankment and provides a direct seepage path to the interior of the dam should the pond water level rise above the invert of the pipe. Measures should be taken to seal off the carrier pipe. PPL Montana has indicated the carrier pipe may not be needed for much longer and can be modified or removed.

6.2.3 Downstream Slope

The downstream slope of the embankment is well-vegetated with grass, which provides some erosion protection (Photo 8), with the exception of the west side which is located in a coal storage area. No signs of major instability were observed along the downstream slope, though some oversteepened areas and rodent holes were observed along the toe on the western side of the embankment (Photos 9 and 10). The oversteepened toe appears to be caused by a cut made to establish a valley drain pipe easement adjacent to the toe of the slope. In addition, numerous elongated sinkholes (up to 1 foot wide, 2 feet long and about 6 inches deep) were observed in this area (Photo 11). PPL Montana personnel indicated the sinkholes were associated with the valley drain pipe alignment at the toe that was placed about 5 feet deep. The pipe was placed in the winter and backfilled in freezing temperatures, suggesting that the sinkholes likely occurred because of volume changes in the thawing backfill material. There is an abandoned manhole associated with a cooling water pipe present at the northwest corner of the downstream toe (Photo 12). The manhole is on the order of 20 feet deep and presents a potential seepage pathway.

Evidence of significant seepage (standing water with vegetation) was observed near the northeast corner of the east cell of the Bottom Ash Ponds (Photo 13). The seepage was at the downstream toe of the east embankment. A box culvert located at the embankment crest is the likely seepage pathway. The water level of the east cell was observed to be at the invert of the box culvert. The box culvert enables discharge pipes to exit the east cell. These pipes are no longer in service and PPL Montana indicated the box culvert can be removed.

6.2.4 Water Surface Elevations and Reservoir Discharge

Surveyed water surface elevations were not available for the Units 1 & 2 Bottom Ash Ponds. The water surface was on the order of 4 feet below the embankment crest at the time of the site visit. No discharge was observed from the bottom ash ponds.

6.3 Units 1 & 2 "A" Pond

Field assessment of the Units 1 & 2 "A" Pond Embankment included walking the embankment crest, upstream slope and downstream slope. We saw no obvious signs of settlement, displacement or adverse seepage that would directly affect the safety of the impoundment. An general photo of the "A" Pond is shown in Photo 14.

6.3.1 Dam Crest

The embankment crest appeared to be in good condition. No signs of cracking of settlement were observed during the assessment. No vegetation was observed on the dam crest (Photo 15).

6.3.2 Upstream Slope

The upstream slope of the embankment is protected by the clay pond liner and appeared to be in satisfactory condition. Some vegetation was observed along the inside slope, but there were no signs of instability (Photo 16).

6.3.3 Downstream Slope

The downstream slope of the embankment is well-vegetated, which provides some erosion protection. No signs of major instability were observed along the downstream slope, though some oversteepened areas, rodent holes and sinkholes associated with the buried valley drain pipe were observed along the toe similar to those described for the Bottom Ash Ponds in Section 6.2.3 (Photo 17).

6.3.4 Water Surface Elevations and Reservoir Discharge

The water surface in the "A" Pond was surveyed at El. 3257.52 in May 2009, which is about 6.5 feet below the crest of the surrounding embankment (El. 3264). No discharge was observed from the "A" Pond.

6.4 Units 1 & 2 STEP Dam

Field assessment of the Units 1 & 2 STEP Dam included walking the embankment crest, upstream slope and downstream slope and observing the emergency spillway. We saw no obvious signs of settlement, displacement or seepage that would directly affect the safety of the dam.

6.4.1 Embankment Crest

The embankment crest appeared to be in good condition. No signs of cracking or settlement were observed during the assessment. The crest does not have surfacing material for the traffic and has minimal vegetation (Photo 18). One concern is that the crest appears to be one or two feet lower than El. 3278 at the right abutment, providing a possible path for water passage at an elevation lower than desired. The low area occurs at the right abutment/dam contact and appears to be associated with an earth cut for an access road.

6.4.2 Upstream Slope

The upstream slope of the dam is generally protected from erosion by an HDPE lining, (Photos 19-20), with the exception of the portion of the dam near the right abutment near Area “D” which is not currently impounding water and is vegetated (Photo 21). The upstream abutment generally appeared to be in good condition. Some moderate erosion rills were observed on the upstream face near the right abutment where surface water collecting on the dam crest runs into unused Cell “D” (Photo 22).

6.4.3 Downstream Slope

The downstream slope of the embankment is well-vegetated, which provides some erosion protection (Photos 23-24). No signs of major instability were observed along the downstream slope. Some erosion rills caused by surface water were observed in the groin near the right abutment (Photo 25). Stormwater flows across the downstream area near the right side of the dam toe due to runoff from a contributing drainage area located southeast of the dam.

6.4.4 Emergency Spillway

The emergency spillway beyond the left abutment of the dam appeared to be in good condition, with no visible deterioration (Photo 26). No pond discharges have ever flowed through the spillway.

6.4.5 Water Surface Elevations and Reservoir Discharge

Water surface elevations in the various STEP pond cells ranged from El. 3256.5 to El. 3264.1 in May 2009, or about 6 to 13.5 feet below the dam crest. No discharge was observed from the STEP pond.

6.5 Units 3 & 4 EHP Pond - Main Dam

Field assessment of the Main Dam at the Units 3 & 4 EHP included walking the embankment crest, upstream slope and downstream slope. We saw no obvious signs of settlement, displacement or seepage that would directly affect the safety of the Main Dam.

6.5.1 Embankment Crest

The embankment crest appeared to be in good condition. No signs of cracking or settlement were observed during the assessment. Because the dam will be raised in the future, the crest is currently wider than its proposed 20 feet. There is a dirt road along the downstream side of the crest, (Photo 27) while the upstream side is vegetated. The dam crest has a length of fill located in a small saddle about 500 feet to the left of the left abutment (Photo 28). This fill should also be considered part of the dam until conditions are documented that indicate it does not serve as part of the dam structure.

6.5.2 Upstream Slope

The upstream slope of the Main Dam is protected by soil cement and appeared to be generally in excellent condition (Photo 29 and 30). Some seepage and erosion was observed at the left abutment groin on the upstream slope. The seepage was located more than 10 feet above the reservoir water level. Ray Womack of Womack Consulting indicated that the seepage originated from perched groundwater within the dam abutment and the adjacent divider dike due to the recent rain (Photo 31).

6.5.3 Downstream Slope

The downstream slope of the embankment is well-vegetated, which provides some erosion protection (Photos 32 and 33). No signs of major instability were observed along the downstream slope. Some animal burrows, including one excavated into the drainage sand at the right downstream groin, and minor erosion rills caused by surface water were observed (Photos 34 and 35). Seepage has been observed in the natural ground downstream of the Main Dam since 2000 (Womack 2009) and studies have shown the flow originates from seepage through the sandstone in the left abutment of the dam. This seep has been referred to as the “552 Seep” in PPL’s documents.

6.5.4 Water Surface Elevations and Reservoir Discharge

Water surface elevations in the various EHP pond cells ranged from El. 3234.5 to El. 3287.1 in May 2009 (the higher elevations are within cells completely surrounded by dikes with crest elevations higher than those of the dams). No discharge was observed from the EHP.

6.6 Units 3 & 4 EHP - Saddle Dam

Field assessment of the Units 3 & 4 EHP Saddle Dam included walking the embankment crest, upstream slope and downstream slope. Significant settlement, displacement, and seepage issues were discussed and observed that would directly affect the safety of the dam for storage of water at its design normal water surface. These issues are not a concern with the current restricted operating level of El. 3237, which is 25 feet below the dam crest.

6.6.1 Embankment Crest

The embankment crest appeared to be in fair condition. Because the dam will be raised in the future, the crest is currently wider than its proposed 20 feet. There is a dirt road along the downstream side of the crest, (Photo 36) while the upstream side is vegetated with a thick stand of sage brush (Photo 37). PPL Montana personnel pointed out healed cracks in areas where cracking and settlement occurred associated with the 1999 seepage event (Photos 38 and 39). Cracks up to about 1 foot wide and several feet deep were originally observed during the inspection by Maxim Technologies in 1999 (Maxim, 2005) and have since been repaired. No new damage was observed beyond that documented in the past.

6.6.2 Upstream Slope

The upstream slope of the dam is protected by soil cement and appeared to be in good condition (Photo 40). Minor vegetation was becoming established on the soil cement in some areas (Photo 41).

6.6.3 Downstream Slope

The downstream slope of the embankment is well-vegetated, which provides some erosion protection (Photos 42 and 43). Several issues were noted during the field assessment of the downstream slope. Near the center of the dam at the downstream toe was an open test pit (Photo 44) that was excavated about 10 years ago to observe the toe drain during the 1999 seepage event. The drain sand and a broken toe drain pipe was visible in the test pit (Photo 45). PPL operates the adjacent cell "G" with a restricted water level below El. 3237 to prevent recurrence of the seepage issue. Some minor surface erosion was also observed along the downstream slope (Photo 46).

Seepage occurred in 1999, 2004 and 2005 at separate locations around the EHP and the seepage flows surfaced 100 to several hundred yards downstream of the dam. After the 1999 incident, the water level behind the Saddle Dam was lowered, and the seepage ceased. The areas where the 1999 seepage discharge occurred was observed (Photo 47). The 2004 and 2005 seepage events occurred to the south and west of the EHP. The south and west sides of the EHP are contained only by the concrete cutoff walls – there is no dam in these areas. The seepage occurred though fractured rock and measures were implemented by PPL to eliminate the source of water in the adjacent cells by first removing the water and then lining the cells or filling them with paste.

6.6.4 Water Surface Elevations and Reservoir Discharge

See the discussion in Section 6.5.4 for EHP Main Dam.

6.7 Field Inspection Observations

6.7.1 Settlement

Settlement cracks have been observed in the crest of the Units 3 & 4 EHP Saddle Dam in conjunction with the 1999 seepage event. The cracks were aligned along the upstream side of the concrete cutoff wall and indicate differential settlement associated with the seepage flows at the contact between the dam embankment and concrete cutoff wall. Internal erosion of embankment material likely occurred due to the seepage flow around the cutoff wall transporting soil particles into the highly permeable baked shale strata. The cracks have healed as a result of precipitation and in-filling and no fresh cracks were observed during the June 2009 assessment. No evidence of settlement was observed in other dams or embankments.

6.7.2 Movement

There was no evidence observed during the inspection to indicate differential movement of project structures, except as noted for the Saddle Dam in Section 6.7.1.

6.7.3 Erosion

There was no significant erosion of the dams or abutments noted during the assessment. Some oversteepening at the toe of the embankment of the on-site ponds was observed, and minor erosion caused by surface water was observed in several locations at the dams at the Units 1 & 2 STEP and the Units 3 & 4 EHP.

6.7.4 Seepage

The only location where uncontrolled seepage was observed during the assessments was on the east side of the Units 1 & 2 Bottom Ash Pond, at the downstream toe of the east embankment. A small pool of standing water with well established grass was visible in this location (Photo 13). The seepage appears to originate from an out-of-service box culvert that penetrates the east embankment near the crest. The box culvert no longer serves a useful purpose and should be removed and the embankment backfilled with engineered fill.

PPL Montana personnel indicated that the Units 3 & 4 Saddle Dam had seepage problems at various locations where springs formed in the foundation rock several hundred feet downstream of the EHP in 1999, 2004 and 2005. Seepage at the 1999 location was not observed during this assessment and the cell "G" reservoir was below the restriction limit El. 3237. The 2004 and 2005 seepage events have been controlled by operational changes and were not visited. See section 6.7.1 for additional discussion of issues associated with the 1999 seepage event.

6.7.5 Leakage

We did not observe water leaking from any of the project structures.

6.7.6 Cracking

There were no new cracks observed in the upstream or downstream slopes or the crests of the dams. Healed cracks in the crest of the Units 3 & 4 EHP Saddle Dam were observed.

6.7.7 Deterioration

No significant deterioration of project structures was observed with exception of the EHP Saddle Dam, which was damaged by the 1999 seepage event and has not been repaired.

6.7.8 Geologic Conditions

The geology of the project features is as described in the prior reports. There have been no studies or events (landslide, earthquake, etc.) that would result in changes to the description of local geologic conditions.

6.7.9 Foundation Deterioration

No signs of foundation deterioration were observed with exception of the EHP Saddle Dam, which was damaged by the 1999 seepage event and has not been repaired.

6.7.10 Condition of Spillway and Outlet Works

The emergency spillway at the Units 1 & 2 STEP Dam appeared to be in good condition. No flows or releases have occurred through the spillway.

6.7.11 Reservoir Rim Stability

The reservoir rims visible from the dam crests did not show any evidence of landslides or shoreline instability that would threaten the safety of the dams.

6.7.12 Uplift Pressures on Structures, Foundations, and Abutments

No evidence of uplift pressure issues was observed with exception of the EHP Main Dam, which has high water levels in the dam abutment rock. These high water levels contribute to seepage through the sandstone that emerge downstream of the dam (the “552 Seep). The high water levels are monitored and are controlled by pumping wells in the abutments to reduce the potential seepage flow.

6.7.13 Other Significant Conditions

None.

7.0 Spillway Adequacy

7.1 Floods of Record

Floods of record have not been evaluated for the ponds at the Colstrip facility.

7.2 Inflow Design Floods

The Units 1 & 2 STEP and the Units 3 & 4 EHP impoundments were designed based on U.S. Army Corps of Engineers (USACE) guidelines that developed a 24-hour probable maximum precipitation (PMP) of 24 inches. Current hydrometeorological guidelines are based on the 72-hour PMP.

Original hydrologic studies for the Units 1 & 2 Bottom Ash Ponds and the "A" Pond are not available. The ponds are designed with 4 feet of freeboard (Portage, 2005) and are not expected to accumulate any significant run-on since they are surrounded by above-grade dikes on all sides. Therefore, the ponds can safely impound the 24-hour PMP with two remaining feet of freeboard, which we expect would be sufficient to store the difference between the 24-hour and the 72-hour PMP. The on-site ponds are therefore considered adequate to store the inflow design floods.

The Units 1 & 2 STEP and the Units 3 & 4 EHP were checked for compliance with the 72-hour PMP (Maxim, 2005). The Units 1 & 2 STEP Dam has been classified as a high hazard dam (Maxim, 2005). The USACE Guidelines for dams requires the spillway on such dams be able to pass the full PMF. The STEP was designed to contain the 100-year flood followed by the PMF associated with the 24-hour PMP, for a total flood volume of 872 acre-feet (Bechtel, 1979). The 2005 Phase I inspection report (Maxim, 2005) indicates that the STEP was independently evaluated using the 72-hour PMP in 1988. The pond was found to be able to hold most of the PMF in this case, while the spillway would safely pass the remaining 501 acre-feet with a maximum discharge of 111 cubic feet per second (cfs) at a depth of 0.8 feet. We reviewed these evaluations and compared them with current hydrometeorological reports, and found the existing STEP Dams and spillway to be able to safely pass the full PMF.

The Units 3 & 4 EHP Dams were classified as Low Hazard dams (Maxim, 2005). However, based on the potential for significant economic/environmental damage and flooding of residences and farmland following a breach the EHP should likely be classified as Significant Hazard and possibly High Hazard. Conservatively assuming that the dams are classified as high hazard, they will be required to pass or safely store the PMF. The EHP was designed to contain the 100-year flood followed by the PMF associated with the 24-hour PMP, for a total flood volume of 1186 acre-feet (Bechtel, 1982) when it is completed to full crest height El. 3290. The 2005 inspection report (Maxim, 2005) indicates that the ponds were independently evaluated using the 72-hour PMP in 1988. The pond was found to be able to hold most of the PMF in this case, with the (future) spillway discharging a maximum of 29 cfs at a depth of 0.4 feet. We reviewed these evaluations and compared them with current hydrometeorological reports, and found the planned EHP dams and spillway to be able to safely pass the PMF based on the full height crest El. 3290.

However, at the time of our assessment, the EHP dams had crests at El. 3262, and construction to their final height, including construction of the emergency spillway, had not been scheduled. The "G" cell that is impounded behind the Saddle Dam currently has a water level at about El. 3234. The cells behind the Main Dam have water levels of El. 3234 (Old Clearwell) and El. 3259 (Area "A"). Cell "G" and the Old Clearwell currently have more than 24 feet of freeboard, which is sufficient to store significantly more than the 24-hour or 72-hour PMP. Area "A" has 3 feet of freeboard and can store the 24-hour PMP, but may be close to overtopping under 72-hour PMP conditions if excess water is not distributed to the adjacent Old Clearwell. In general, the Main and Saddle Dams are considered adequate to store the design floods at their current height, but the capacity of Area "A" to store the 72-hour PMP should be evaluated. Given the complications associated with the 1999 seepage through the Saddle Dam, water levels should be maintained at, or below, the restricted level of El. 3237 to the extent possible until remedial measures are implemented.

7.2.1 Determination of the PMF

The PMF based on the 24-hour PMP is 24 inches per hour as determined in the design reports for the STEP and EHP Dams. The dams have been previously checked and found adequate to safely pass the 72-hour PMP.

7.2.2 Freeboard Adequacy

Freeboard is adequate at all facilities.

7.2.3 Dam Break Analysis

Dam break analyses were not performed for the Units 1 & 2 Bottom Ash Pond or the "A" Pond embankments. Due to PPL Montana's classification of these ponds as Significant Hazard and the potential concern cited for loss of life, a dam break analyses and inundation mapping for these ponds should be performed.

Dam break analyses and inundation maps are available for the Units 1 & 2 STEP Dam and the Units 3 & 4 EHP Dams (Maxim, 2005 and 2008). The inundation map for the STEP dam revealed that a breach of this dam would cause flooding of nearby residences, businesses, a highway and a railroad. The inundation mapping for the EHP dams shows that the flood wave would travel many miles down Cow Creek and flooding isolated farm buildings and residences. The inundation maps were reviewed for this assessment and are considered adequate. However, the EHP inundation map evaluation (Maxim, 2008) did not focus on the potential for significant economic and particularly, environmental damage associated with a breach. Our brief review indicated that the potential economic/environmental damage could be significant and that the EHP should be classified as Significant Hazard at a minimum. The flooding of habitable structures and residences downstream should also be further evaluated to determine the potential for loss of life under Federal guidelines and the associated hazard classification.

7.3 Spillway Rating Curves

Spillway rating curves for the STEP Dam emergency spillway was not provided. The EHP does not have an emergency spillway.

7.4 Evaluation

Upon review of the design floods developed by Bechtel and re-evaluated by Maxim, the emergency spillway discharge capacity at the STEP Dam appears to be adequate to safely pass the regulatory design floods based on the hazard classifications for the dams. The EHP water levels should continue to be restricted unless remedial measures are taken to repair the Saddle Dam or the Saddle Dam is used for storing paste exclusively and the documentation is modified to reflect this use. Design PMP and inflow flood information for the small Units 1 & 2 Bottom Ash and "A" ponds is not available, but based on dam crest elevations and water storage elevations these ponds appear to have sufficient freeboard to store the PMF for this region.

8.0 Structural Stability

8.1 Visual Observations

No visible signs of instability were evident associated with the any of the dams or embankments during the June 2009 site assessments.

8.2 Discussion of Stability Analysis

8.2.1 Units 1 & 2 Bottom Ash Ponds and “A” Pond

Slope stability analyses and inspection reports were not available for the on-site ponds. We performed preliminary stability analyses on these embankments using the limit equilibrium computer program SLOPE/W. These stability analyses were performed with the most current and relevant geometry information available to us.

We based the embankment geometry for the slope stability analyses on the cross sections shown in the Bottom and Fly Ash Sections drawing (Exhibit 1). Analyses were performed for Sections A, B and C for the cases of steady state seepage and seismic loading. Soil parameters were assumed to be the same as those used for design of the STEP Dam (Bechtel, 1979). Bedrock depth was estimated from the Portage, 2005 report on design of the “B” Bottom Ash Pond and was assumed to be at El. 3217. Piezometric surfaces were estimated using conservative assumptions. The soil material properties adapted from the STEP design are listed in Table 3.

Table 3: Material Properties used for Slope Stability Analyses of Units 1 & 2 Bottom Ash and “A” Pond Embankments

Material	Drained Friction Angle, ϕ’ (degrees)	Drained Cohesion, c’ (psf)	Undrained Friction Angle, ϕ (degrees)	Undrained Cohesion, c (psf)	Unit Weight
Random Fill (Same as Shell for STEP Dam)	33	0	22.5	750	120
Core	33.5	0	13	1000	120
Foundation	32	0	17.5	700	120
Bedrock	0	4000	0	4000	130

Graphic results of stability analyses are shown in Appendix E. Factors of safety are discussed and summarized below in Section 8.3.

8.2.2 Units 1 & 2 STEP Dam

The results of slope stability analyses performed for the design of the STEP dam are reported in the Bechtel 1979 "Second Stage Evaporation Pond Design Report." The analyses were performed using the Simplified Bishop Method of Slices with the computer program SLOPE developed at MIT. Load cases analyzed included Normal Pool (El. 3270, referred to as "Maximum Pool" in the design report), Normal Pool with Seismic Loading (0.05g) and End of Construction (which is no longer of concern since the dam has been in place for more than twenty years). In the 1979 design, the rapid drawdown condition was not modeled based on reasoning there is no low-level outlet to rapidly drain the reservoir. Both the upstream and downstream slopes were analyzed and the 2005 inspection report (Maxim, 2005) indicates that these analyses were checked independently in 1988 and factors of safety were found to meet or exceed Federal Energy Regulatory Commission (FERC) requirements. The 1988 inspection report was not made available to GEI.

The material properties used in the stability modeling were based on laboratory testing of site-specific materials with some conservative assumptions. Where laboratory test data were scarce, data from the First Stage Evaporation Pond dam design laboratory tests were included in parameter development. Information on the phreatic surface assumed within the dam was not available in the 1979 report. All factors of safety calculated were higher than those required by the FERC.

The stability analyses included in the 1979 report were reviewed. The loading conditions used in the previous analyses have not changed and these analyses are considered adequate. In addition, rapid drawdown should be evaluated to examine the potential effects of a dam breach or other sudden leakage event.

8.2.3 Units 3 & 4 Main Dam

The results of slope stability analyses performed for the design of the dam are reported in the Effluent Holding Pond Design Report (Bechtel, 1982) The analyses were performed using the Simplified Bishop Method of Slices with the computer program SLOPE developed at MIT for the final configuration of the Main Dam with crest at El. 3290. Load cases analyzed included Normal Pool (El. 3280, referred to as "Maximum Pool" in the design report), Normal Pool with Seismic Loading (0.05g) and End of Construction (which is no longer of concern since the dam has been in place for more than twenty years). In the 1982 design, the rapid drawdown condition was not modeled based on reasoning that there is no low-level outlet from which the pond can be rapidly drained. Both the upstream and downstream slopes were analyzed. Information on the phreatic surface assumed within the dam was not available in the 1983 report. The 2005 inspection report (Maxim, 2005) indicates that these analyses were checked independently in 1988 and factors of safety were found to meet or exceed the minimum factors of safety required by FERC. The 1988 inspection report was not made available to GEI.

The material properties used in the stability modeling were based on laboratory testing of site-specific materials with some conservative assumptions. Design strength parameters for the shell material were based on similar materials used for the Units 1 & 2 STEP Dam, located four miles away. Chen and Associates performed a geotechnical exploration of the shell material in 1989 and confirmed that the shell material was at least as strong as was assumed in the design report.

Several boreholes were drilled and completed as observation wells or piezometers in the dam and abutments in 2001. Stability analyses were subsequently performed by Hydrometrics to evaluate the stability of the dam in its current configuration with crest El. 3262 based on the pore pressure information from one piezometer in the dam core and also for an assumed phreatic surface to model a case where the internal drains malfunctioned and excess pore water pressures built up beneath the embankment. Analyses were performed for two cross sections assuming both circular and block failure surfaces. For the case based on the piezometer data, the factors of safety were found to meet or exceed the minimum factors of safety required by FERC in the 2001 study. For the case based on the assumed excess water pressures, the factors of safety were found to be as low as 1.23 and did not meet the minimum factors of safety of 1.5 required by FERC. This analysis was considered to be very conservative in the 2001 study. In 2009, piezometer data from 2007 was used to re-run the slope stability analyses with the dam crest at El. 3262, and a factors of safety of 1.5 was calculated, which meets the minimum factors of safety required by FERC. The 2009 report concluded that the chimney drain was functioning as designed (Womack, 2009).

The stability analyses discussed above were reviewed for this assessment. The 2001 and 2009 analyses indicate conditions are present for a potentially significant seepage issue where seepage pressures in the sandstone can bypass the cutoff wall and act on the downstream embankment shell. This potential is illustrated by the available data that shows the pressure head in the abutment sandstone is about 5 feet higher than in the adjacent dam core. The 2001 analysis attempts to model higher pore pressures in the dam core and downstream shell, however the basis for the model is not well established because there is a lack of pore pressure data at those locations. There is only one piezometer installed in the dam and one in each abutment. The analysis of the potential seepage pressure case does not appear to be complete and the instrument data needed to perform the analysis with increased certainty is not available.

8.2.4 Units 3 & 4 EHP Saddle Dam

The results of slope stability analyses performed for the design of the dam are reported in the Bechtel 1983 “Effluent Holding Pond Design Report.” The analyses were performed using the Simplified Bishop Method of Slices with the computer program SLOPE developed at MIT for the dam in its final configuration with crest El. 3290. The load cases analyzed are the same as those discussed above for the Main Dam. Both the upstream and downstream slopes were analyzed and the 2005 inspection report (Maxim, 2005) indicates that these analyses were checked independently in 1988 and factors of safety were found to meet or exceed the minimum factors of safety required by FERC. The 1988 inspection report was not made available to GEI.

The material properties used in the stability modeling were based on laboratory testing of site-specific materials with some conservative assumptions. Where laboratory test data were scarce, data from the First Stage Evaporation Pond dam design laboratory tests were included in parameter development. Information on the phreatic surface assumed within the dam was not available in the 1979 report. All calculated factors of safety were higher than those required by FERC.

The stability analyses included in the 1979 report were reviewed. The loading conditions used in the previous analyses have not changed and these analyses are considered adequate for the dam at its final height.

8.3 Factors of Safety

8.3.1 Units 1 & 2 Bottom Ash and “A” Dams

Our check analyses for the Bottom Ash and “A” Pond embankments resulted in a calculated factor of safety ranging from 1.1 to 1.5 for the steady state seepage load case and from 1.7 to 2.1 for the seismic load case depending on the location of the cross section.

For the section of embankment near the Bottom Ash Pond, the factors of safety for steady seepage case and seismic case were 1.5 and 2.1, which meet or exceed the criteria accepted by FERC. The west embankment cross-sections near the Bottom Ash Pond includes a clay core and cutoff trench.

Further south, the west embankment impounds Pond “A”. At this location the embankment transitions from including a clay core without cutoff trench at its north end to not having a core at its south end. For the south section of the west embankment for the “A” Pond, the minimum calculated factors of safety for steady seepage case and seismic case were 1.1 and 1.7. In this case, the factor of safety for steady seepage does not meet the minimum criteria accepted by FERC of 1.5. Factors of safety calculated for the seismic load case exceed the minimum criteria of 1.1 accepted by FERC.

The low factor of safety of 1.1 for the embankment without core indicates its stability is marginal for the assigned strength and phreatic surface conditions. We recommend performing additional analyses using site-specific strength data for the embankment soil and any appropriate adjustments to the phreatic surface to verify the stability condition and identify if remedial measures are warranted. The embankment has performed adequately since its construction in the 1970s but also depends on the integrity of the clay lining, which was neglected in our analysis.

8.3.2 Units 1 & 2 STEP Dam

We reviewed the calculated factors of safety for the embankment contained in the Bechtel 1979 draft report. This report show factors of safety ranging from 1.6 to 2.0 for the steady seepage load case and from 1.3 to 1.6 for the seismic load case. These factors of safety exceed the minimum factors of safety required by FERC as presented in Table 3.

8.3.3 Units 3 & 4 EHP Main Dam

We reviewed the factors of safety for the embankment contained in the various reports completed for the EHP Main Dam. The original design report (Bechtel, 1982) indicates that factors of safety range from 1.8 to 2.0 for steady state seepage and from 1.3 to 1.7 for seismic loading for the dam at its final height of El. 3290. In a 2001 stability analysis, which used pore water pressure information from a recently-installed piezometer, steady state seepage factors of safety for the dam at its current crest El. 3262 ranged from 1.6 to 1.9 for existing pore water pressure conditions and from 1.2 to 1.6 for assumed higher pore water pressures downstream of the core resulting to model a potential malfunction of the internal drains (Hydrometrics, 2001). The 2009 Stability Analysis Review Update for the dam at its current crest El. 3262 indicates factors of safety of 1.5 using pore water pressure conditions as measured in the piezometer in 2007 (Womack & Associates, 2009). These factors of safety exceed the minimum factors of safety required by FERC as presented in Table 3. However, as discussed below, the level of conservatism realized by the 2001 and 2009 analyses is not certain and would benefit from additional pore pressure measurements within the downstream shell and core of the dam and abutment sandstone.

Though not stated outright, the 2001 and 2009 stability analyses appear to be studies of the potential for high pore pressures to be introduced into the dam embankment from seepage in the sandstone strata that is present in the abutments. In the Station 19+00 cross section shown in Figure 2 from the report by Hydrometrics (2001), the sandstone stratum extends above the bottom of the dam core, and extends through the abutments of the dam beyond the end of the core. The sandstone is known to carry seepage from the reservoir and is known to have fractured zones with higher permeability as discussed in the study of the "552 seep" downstream. The sandstone could enable high seepage pressures from the reservoir to come into contact with the downstream embankment, which would worsen existing seepage conditions and potentially contribute to instability of the dam.

The measured piezometer water levels in the sandstone are only 5 feet higher than those measured in piezometer 636P, which is located near the base of the core and just downstream of the cutoff wall. The potential for high pore pressures to exist in the sandstone and be introduced into the dam embankment downstream of the core and cutoff wall is a significant dam safety concern. The 2001 and 2009 analyses are based on information from only one functioning piezometer in the dam core (636P), and does not include sufficient information about pore pressure conditions elsewhere in the dam, particularly in the downstream shell. The initial data from piezometer 637P indicates water pressures may be 10 feet higher than recorded in 636P. As a result, the level of conservatism presented in the 2001 and 2009 analyses is not certain. We recommend additional analysis include seepage modeling of the dam and abutments and additional pore pressure measurements obtained within the downstream shell and core of the dam and abutment sandstone.

8.3.4 Units 3 & 4 EHP Saddle Dam

We reviewed the computed factors of safety for the embankment contained in the Bechtel 1982 draft report. This report show factors of safety ranging from 1.7 to 1.8 for the steady state seepage load case and from 1.4 to 1.5 for the seismic load case for the dam at its final height with crest El. 3290. These factors of safety exceed the minimum factors of safety required by FERC as presented in Table 4.

Stability analyses performed for the Saddle Dam appear to adequately address critical dam sections with exception that the compromised seepage control measures have not been addressed. The analyses presented in the original design report all meet the minimum required factor of safety criteria according to FERC guidance. However, due to the observation of seepage downstream of the toe of the saddle dam and the lowering of water levels that was required to stop the seepage, we recommend that stability be re-evaluated incorporating any new geotechnical or hydrologic information. In particular, the revised analyses should include conservative assumptions regarding the capacity of seepage control measures if the seepage control features have not been satisfactorily repaired. If the dam will only be used to impound paste for the rest of its life, an appropriate model should be used to reflect the potential for reduced seepage.

8.3.5 Summary

We compare the reported calculated factors of safety for the STEP and EHP dams to minimum required factors of safety in accordance with FERC guidelines in Table 4. Values shown are the minimum factor of safety from the most recently calculated analyses. The dams at the Colstrip facility are not regulated under the Montana Dam Safety Act, so those guidelines are not included. The Montana Dam Safety guidelines refer to the USACE guidelines, which are the same as, or less conservative than, the FERC guidelines shown in Table 4.

Table 4: Stability Factors of Safety for Colstrip Facility Dams and Guidance Values

Loading Condition	Min. Calculated FOS, Bottom Ash and "A" Pond Embankments	Min. Calculated FOS, STEP Dam (Crest El. 3290)	Min. Calculated FOS, EHP Main Dam (Crest El. 3290)	Min. Calculated FOS, EHP Main Dam (Crest El. 3262)	Min. Calculated FOS, EHP Saddle Dam	Min. Required FOS (FERC)
End of Construction	NA	2.0	2.0	NA	1.8	1.3
Full Reservoir – Steady Seepage	1.1 (Pond "A")	1.6	1.8	1.5	1.7	1.5
Full Reservoir – SS with Earthquake (0.05g)	1.7	1.3	1.3	NA	1.4	1.0

As indicated in Table 4, the calculated factors of safety for static and seismic conditions meet or exceed the minimum required FERC guidelines except for Pond "A" steady seepage, which has a marginal factor of safety of 1.1 as analyzed. Also, the original analyses for EHP Main Dam and Saddle Dams were for the crest at El. 3290, which is 28 feet higher than the current crest.

The potential for high pore pressures to exist in the sandstone and be introduced into the dam embankment downstream of the core and cutoff wall is a significant dam safety concern based on the available instrument data. The 2001 and 2009 stability analyses do not include sufficient information about pore pressure conditions elsewhere in the dam, particularly in the downstream shell and, as a result, may not be sufficiently conservative. We recommend performing additional analyses that includes seepage modeling of the dam and abutments, obtaining additional pore pressure measurements within the downstream shell and core of the dam and abutment sandstone, and validating the model with measured seepage flow rates collected by the internal drains.

8.4 Seismic Stability - Liquefaction Potential

The liquefaction potential at the various project features was not evaluated in the design studies because saturated granular soils that are potentially liquefiable are not present in the dam embankment and foundation.

9.0 Adequacy of Maintenance and Methods of Operation

9.1 Procedures

There are no written Standard Operating Procedures for the Colstrip impoundments. The operations of the impoundments are largely determined by the water recycle needs of the power plant.

The bottom ash and fly ash from Units 1 and 2 are pumped as slurries to the on-site ponds just south of the plant. Partial settling of particulates occurs in these ponds and the remaining clearwater is returned to the plant. A minor amount of the bottom ash is reclaimed from the on-site ponds and used for construction of roads and dikes on site. The remaining fly ash and bottom ash is pumped to the Units 1 & 2 STEP for final settlement and storage. A new paste plant located at the STEP will process the fly ash beginning in 2010.

The bottom ash from Units 3 & 4 is pumped to the on-site bottom ash ponds to the east of the plant for temporary storage, and ultimately to the Units 3 & 4 EHP for final settlement and storage. The fly ash from Units 3 & 4 is pumped directly to the Units 3 & 4 EHP paste plant prior to being pumped into the pond. The fly ash slurry is processed in the paste plant and pumped into the pond as paste with 68 percent solids (versus about 15 percent solids for typical scrubber slurry).

9.2 Maintenance of Dams

Maintenance of the dams and embankments at the Colstrip facility is performed or subcontracted by PPL Montana staff. Inspections are made annually by PPL engineers and approximately every five years by outside consulting engineers.

9.3 Surveillance

PPL Montana staff is responsible for the surveillance of the dams and appurtenant facilities. Monitoring of the dams instrumentation currently occurs monthly. The main power plant is manned 24 hours a day and operators can respond to potential emergency situation at the dams. There are no automatic warning systems for the dams.

10.0 Emergency Action Plan

The Montana State Dam Safety Program, requires that all dams classified as “high hazard” have an emergency action plan. It is our understanding that the Units 1 & 2 STEP Dam and the Units 3 & 4 EHP Dams have emergency action plans with inundation maps which are on file at the Colstrip plant. We recommend a breach inundation map be developed for the common Units 1 & 2 Bottom Ash and “A” Ponds to enable evaluation of potential downstream hazards for these impoundments. The EAPs were not reviewed as part of the assessment.

11.0 Conclusions

11.1 Assessment of Dams

11.1.1 Units 1 & 2 Bottom Ash Pond Embankments

- The 24-inch HDPE pipe protruding from the interior southwest corner of the westernmost cell provides a direct seepage path to the interior of the dam.
- Oversteepened areas and rodent holes were observed along the downstream toe of the west embankment, as well as small sinkholes associated with the buried valley drain pipe alignment along the downstream toe.
- An abandoned manhole at the downstream toe of the northwest corner of the impoundment presents a potential seepage path.
- Evidence of seepage (standing water with vegetation) was observed at the downstream toe of the east cell. A box culvert near the crest of the embankment is the likely seepage pathway. The box culvert is no longer in service.

11.1.2 Units 1 & 2 "A" Pond Embankment

- Oversteepened areas, rodent holes and sinkholes associated with the buried valley drain pipe were observed along the downstream toe of the west embankment similar to those described above for the Bottom Ash Ponds.

11.1.3 Units 1 & 2 STEP Dam

- The crest of the dam is lower than El. 3278 at the right abutment, providing a possible flow path resulting in concentrated flows at high reservoir elevations.
- Some erosion rills were observed on the upstream slope near the right abutment and on the downstream slope in the groin near the right abutment.

11.1.4 Units 3 & 4 EHP Main Dam

- The small saddle fill located about 500 feet left of the left abutment is not currently considered part of the Main Dam, and may function as part of the Main Dam.

- Several animal burrows, including one that exposed drainage sand in the right groin drain, and minor erosion rills caused by surface water were observed on the downstream face of the dam.

11.1.5 Units 3 & 4 EHP Saddle Dam

- An old test pit remains open at the downstream toe of the saddle dam. Drain sand and a broken toe drain pipe are exposed in the test pit.

- Minor surface erosion was observed on the downstream slope.

- Seepage occurred in 1999 at a location downstream of the dam. After this incident, the water level in cell “G” behind the Saddle Dam was lowered, and the seepage ceased. The water level in cell “G” remains restricted to El. 3237, but rehabilitation of the Saddle Dam has not been performed.

- Seepage events occurred though fractured rock to the south and west of the EHP in 2004 and 2005. The south and west sides of the EHP are contained only by the concrete cutoff walls – there is no dam in these areas. The seepage has been addressed by PPL by eliminating the source of water in the adjacent cells by first removing the water and then lining the cells or filling them with paste.

11.1.6 Stability Analysis (Adequacy of Factors of Safety)

We performed check stability analyses of the Units 1 & 2 Bottom Ash and “A” Pond west embankment using soil strength parameters from the STEP Dam report. Factors of safety were found to be below FERC requirements for steady state seepage for the south part of the west embankment that impounds the “A” Pond and indicate marginal stability conditions. The analysis conservatively neglects the contribution of the clay lining in Pond “A” to reducing seepage conditions.

The stability analyses that have been performed for the Units 1 & 2 STEP Dam appear to adequately address critical sections in general, and the analyses meet the minimum required factor of safety criteria according to FERC guidance.

Stability analysis of the Units 3 & 4 EHP Main Dam does not fully address the potential for high pore pressures to be introduced downstream of the core by seepage through the sandstone in the abutments, which presents a significant dam safety issue. Analyses performed to date have been based on insufficient data to fully understand the pore pressure conditions within the dam and the effectiveness of the internal drain system. Additional piezometer instruments are needed in the downstream embankment shell and downstream abutments. Additional seepage analyses are needed to model the phreatic surface and relationship to the abutment sandstone. Measurements of flow rates collected by the internal drains are needed to verify their function and the to calibrate the seepage models.

Stability and seepage should be re-evaluated for the Saddle Dam if there are operating conditions that would require the dam to impound liquid. The Saddle Dam should continue to the operated with the El. 3237 reservoir restriction. The rapid drawdown load case has not been analyzed for either the STEP or EHP dams.

11.1.7 Stress Evaluation

Stress evaluation is not applicable to the dams at the Colstrip facility because there are no structural elements or buildings that would warrant a stress evaluation.

11.1.8 Spillway Adequacy

The emergency spillway discharge capacity at the STEP Dam appears to be adequate to safely pass the regulatory design floods based on its High Hazard classification. The EHP does not have an emergency spillway, but appears to have sufficient capacity to store the regulatory design flood even if the dam were classified as High Hazard. Design flood information for the on-site Units 1 & 2 Bottom Ash and “A” Ponds is not available, but based on the dam crest elevations and water storage elevations these ponds appear to have sufficient freeboard to store the PMF for this region.

11.2 Adequacy of Instrumentation and Monitoring of Instrumentation

The instrumentation in the dams and embankments is inadequate. There are no piezometers, or movement monuments in the Units 1 & 2 STEP Dam. The single electric piezometer in the Units 3 & 4 Main Dam and two piezometers in the abutment sandstone are inadequate to develop full understanding of the pore pressures within the dam and abutment, which present a potentially significant dam safety issue. There are no piezometers for the Units 1 & 2 Bottom Ash and “A” Pond embankments, particularly the west embankment which likely has areas with less than required static factor of safety and may have marginal stability.

11.3 Adequacy of Maintenance and Surveillance

The dams and embankments and the PPL Montana Colstrip facility have satisfactory maintenance and surveillance programs. Significant seepage problems have been observed and remedied in the past. However, more routine maintenance to address surface erosion, rodent burrows, and to backfill/repair excavations (Saddle Dam test pit) is not addressed promptly.

11.4 Hazard Classification

The Units 1 & 2 Bottom Ash Ponds and “A” Pond were classified by PPL as “Significant Hazard” due to the vicinity of residences and Arnell’s Creek and the potential for loss of life in the event of a breach. EPA hazard classification states that any dam whose “failure or misoperation will probably cause loss of human life” should be classified as a High Hazard dam. We concur that the minimum appropriate classification for these ponds is Significant Hazard. The potential hazards associated with these on-site ponds should be re-examined to determine the appropriate classification.

The Units 1 & 2 STEP Dam was classified (Maxim, 2005) as a High Hazard dam due to the high potential for loss of life and extensive property damage in the event of a failure. This hazard classification is considered appropriate.

The Units 3 & 4 EHP Dam was classified (Maxim, 2005) as Low Hazard dams based on interpreted minimal potential for damage and dissipation of the flood wave in a broad floodplain such that “slow flooding of residences” is possible. The EPA hazard potential classification indicates that Low Hazard Potential structures result in “low economic and/or environmental losses,” while those with Significant Hazard Potential are “those dams where failure or misoperation…can cause economic loss, environmental damage, disruption of lifeline facilities…”. We believe that the minimum appropriate classification for this dam is Significant Hazard based on potential for economic loss and environmental damage and that the dam may need to be classified as High Hazard based on the potential for loss of life due to flooding of inhabited structures and residences. We recommend that the hazard classification for this dam be re-evaluated to determine the appropriate classification.

12.0 Recommendations

12.1 Corrective Measures for the Structures

12.1.1 Units 1 & 2 Bottom Ash Pond Embankments

1. A check slope stability analysis was performed by GEI because an existing analysis was not available. The check stability analysis indicates the west embankment of the Bottom Ash Ponds meets the minimum required factors of safety in accordance with the FERC. However, we recommend that slope stability analyses be performed and documented for these embankments based on site-specific information.

2. Modify the 24-inch HDPE carrier pipe in the southwest corner of the west cell to prevent a potential seepage path at higher reservoir elevations through the HDPE lining to the interior of the embankment.

3. Remove the out-of-service box culvert located near the embankment crest on the east cell and backfill with engineered fill.

4. Implement rodent control measures on the downstream slope of the embankment to reduce the potential for shortened seepage pathways through the burrows.

5. Place engineered fill and regrade the downstream toe of the embankment to eliminate oversteepened slopes.

6. Remove and backfill the out-of-service manhole at the downstream toe of the northwest corner of the west cell to eliminate this potential seepage pathway.

7. Install piezometers to monitor water pressures in the embankment and foundation. Collect and evaluate data at least twice per year.

12.1.2 Units 1 & 2 "A" Pond Embankments

1. Slope stability check analyses performed by GEI indicate the south part of the west embankment of the "A" Pond has less than required, and potentially marginal, stability under static steady seepage conditions. This part of the embankment does not have a clay core. Perform additional stability analyses of these embankments using site-specific soil strength information, install piezometers to obtain pore pressure information, and closely monitor stability and seepage conditions.

2. Implement rodent control measures on the downstream slope of the embankment to reduce the potential for shortened seepage pathways through the burrows.

3. Fill and regrade the oversteepened areas at the downstream toe of the embankment.

4. Install piezometers to monitor water pressures in the embankment and foundation. Collect and evaluate data at least twice per year.

12.1.3 Units 1 & 2 STEP Dam

1. Correct the low area of the dam crest at the right abutment by placing engineered fill.

2. Repair the erosion on the upstream slope near the right groin. Correct surface water run-on to eliminate the water source for future erosion. Repair the minor surface erosion on the upstream and downstream slopes of the STEP Dam.

3. Install piezometers and movement monuments in the dam to monitor water pressures and displacement. Install a means of measuring seepage flow collected by the internal drain system. Collect and evaluate data at least twice per year.

12.1.4 Units 3 & 4 EHP Main Dam

1. Install additional instrumentation in the dam and sandstone layer in the dam abutments. Some of these instruments should obtain data in the downstream shell and in the abutment at a location downstream of the core. Collect and evaluate data at least twice per year.

2. Perform seepage and stability analyses to develop understanding of the potentially critical abutment seepage condition in the sandstone layer.

3. Continue to monitor water levels in the dam and abutments and the associated seep that surfaces downstream of the Main Dam and the 1999 seep area downstream of the Saddle Dam.

4. Evaluate and document whether the small saddle fill located about 500 feet left of the left abutment functions as part of the Main Dam. If determined to be part of the Main Dam, the fill should be analyzed for slope stability and inspected regularly like other portions of the dam.

5. Implement rodent control measures on the downstream slope of the dam to reduce the potential for seepage through burrows.

6. Continue to monitor and repair minor surface erosion rills on the downstream slope of the Main Dam.

7. Maintain the free water level restriction in the Old Clearwell at El. 3238.

12.1.5 Units 3 & 4 EHP Saddle Dam

1. The 1999 seepage event that resulted in internal erosion of the Saddle Dam embankment and core was addressed by lowering and restricting the water level behind the dam, but no repairs were made to the dam. The water level restriction should be continued and storage for the appropriate inflow design flood maintained. The dam is not considered safe if water levels are allowed to rise significantly above El. 3237 because the potential for internal seepage erosion remains. However, the EHP ponds impounded by the Saddle Dam are currently being filled with paste consisting of 68 percent solids that cures to a solid. Filling the ponds with paste will greatly reduce or eliminate seepage pressures on the dams and an engineering analysis of the potential to store paste above the restriction level should be documented.

2. Backfill the test pit located on the downstream slope of the dam after repairing the damaged toe drain pipe and restoring the granular drain materials.

3. Continue to monitor and repair minor surface erosion rills on the downstream slope of the Saddle Dam.

4. Maintain the free water level restriction in the "G" cell at El. 3237.

12.2 Corrective Measures Required for Maintenance and Surveillance Procedures

None.

12.3 Corrective Measures Required for the Methods of Operation of the Project Works

None.

12.4 Any New or Additional Monitoring Instruments, Periodic Observations, or Other Methods of Monitoring Project Works or Conditions That May Be Required

The visual inspections and the instrumentation monitoring plan currently in place for the impoundments generally appears to be adequate.

The instrumentation for the dams is inadequate. Install additional instruments in the Units 3 & 4 Main Dam to enable engineering evaluation of water pressures within the core and downstream shell and within the abutment sandstone layer downstream of the core. Install instruments for monitoring water pressures within the Units 1 & 2 STEP dam embankment and in the abutments, particularly the left abutment that is protected by the upstream soil blanket. Install instruments for monitoring water pressures within the Units 1 & 2 Bottom Ash and "A" Pond embankments.

12.5 Acknowledgement of Assessment

I acknowledge that the management unit(s) referenced herein was personally inspected by me and was found to be in the following condition **(select one only):**

SATISFACTORY

FAIR

POOR

UNSATISFACTORY

SATISFACTORY
No existing or potential management unit safety deficiencies are recognized. Acceptable performance is expected under all applicable loading conditions (static, hydrologic, seismic) in accordance with the applicable criteria. Minor maintenance items may be required.

FAIR
Acceptable performance is expected under all required loading conditions (static, hydrologic, seismic) in accordance with the applicable safety regulatory criteria. Minor deficiencies may exist that require remedial action and/or secondary studies or investigations

POOR
A management unit safety deficiency is recognized for any required loading condition (static, hydrologic, seismic) in accordance with the applicable dam safety regulatory criteria. Remedial action is necessary. POOR also applies when further critical studies or investigations are needed to identify any potential dam safety deficiencies.

UNSATISFACTORY
Considered unsafe. A dam safety deficiency is recognized that requires immediate or emergency remedial action for problem resolution. Reservoir restrictions may be necessary.

I acknowledge that the management unit referenced herein:

Has been assessed on June 2 & 3, 2009 (date)

Signature:________________________________

List of Participants:

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Mary Nodine, P.E.	GEI Consultants, Inc.
Joe Byron	Environmental Protection Agency
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Mike Holzwarth	PPL Montana
Steve Christian	PPL Montana
Ray Womack	Womack & Associates, Inc.
Iver Johnson	Montana Department of Environmental Quality

13.0 References

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Womack & Associates, Inc., 2009, 3/4 EHP Main Dams Observations and Stability Review Update, May.

Figures and Exhibits

Appendix A

Instrumentation

Appendix B

Inspection Checklists

June 2 and 3, 2009

Appendix C

Inspection Photographs

June 2 and 3, 2009

Appendix D

Reply to Request for Information Under Section 104(e)

Appendix E

Stability Evaluation for Units 1 & 2 Bottom Ash and “A” Ponds